MODELLING AND ANALYSIS OF MULTI-STOREY BUILDINGS DESIGNED TO PRINCIPLES OF DUCTILITY AND DAMAGE AVOIDANCE

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ABSTRACT: The seismic performance of two typical 10 storey reinforced concrete moment-resisting frame buildings detailed to different design philosophies is examined. Two and three dimensional computational models are used to examine seismic performance of these two buildings with conventional seismic ductile design and damage avoidance design (DAD) details. The models are calibrated against experimental results. Using a suite of 20 earthquakes, Incremental Dynamic Analysis (IDA) is conducted and the responses of the buildings to these earthquakes are interpreted probabilistically. Expected damage and loss modes due to different levels of seismic hazards are developed. Comparisons are made between: (1) the response of the two and three dimensional models to bi-directional earthquake excitations, considering interaction between moments and forces in 3D elements; and (2) the performance of the two different buildings designed according to the different design philosophies. Results indicate that for certain structures and earthquake excitations the response of the 2D model in orthogonal directions can be super-imposed to accurately approximate the 3D response. However, in other cases the interaction between orthogonal directions leads to increased engineering demands that are under-estimated by 2D modelling. The DAD building is observed to have significantly superior performance compared to the ductile building.

KEYWORDS: Incremental Dynamic Analysis (IDA), fragility, 3-dimensional modelling, Damage Avoidance Design (DAD), ductility design

1. INTRODUCTION

Current design practice of typical reinforced concrete multi-story buildings in New Zealand involves hand calculation methods to determine member sizes, reinforcement layouts and any other special detailing (if needed), which is then checked by performing a computational analysis of the building subjected to an earthquake ground motion. Customary practice is to consider planar (2D) models of a single frame and then extrapolate the predicted response to infer the behaviour of the structural system as a whole. In this research, the response of a ten-storey frame building designed to principles of Damage Avoidance Design (DAD) and conventional capacity design with ductile detailing philosophies is investigated. A computational technique known as Incremental Dynamic Analysis (IDA) [1] is used to consider the structural response of the building in two and three dimensional computation. User-defined damage states are then used to convert the IDA curves into fragility curves and assess the performance of the two design philosophies probabilistically.

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2. COMPUTATIONAL MODEL DEVELOPMENT AND CALIBRATION

Initially, an analytical model of a 3-dimensional (3D) beam-column joint subassembly designed for damage avoidance was constructed using the finite element program Ruaumoko3D [2]. The beams and columns were represented using elastic Giberson beam frame elements. The behaviour of the rocking joint was described using two springs of zero length in parallel (Figure 1). The springs had tri-linear-elastic and elasto-plastic hysteretic behaviours representing the gap opening and supplemental energy dissipating system respectively. Interaction between forces in orthogonal directions was considered in the springs using an elliptical yield surface. No allowance for the yeilding of the post-tensioning tendons was made. The parameters of the springs representing the rocking joints were calibrated based on preliminary quasi-static tests up to 3% drift on a rocking 3-dimensional beam-column subassembly designed for damage avoidance [3]. A full 3D computational model of the ten-storey prototype building was then developed based on the calibrated parameters. The prototype building consisted of a ten storey two-way moment-resisting frame with three bays in each orthogonal direction and contained rocking connections at beam-column joints on the first six stories, and rocking columns at the base in the ground floor and beneath the seventh floor. Connections from the seventh to the tenth floor were monolithic. The flooring system was a one-way precast system (i.e. hollowcore units or double tees), therefore only one of the two orthogonal frames carried gravity loads.

The ductile prototype building was the same as the ten-storey DAD prototype building except that it was designed using capacity design principles to NZS3101 [4]. Computational modelling of this building was performed in Ruaumoko3D using concrete beam-column frame elements with an M-N yield interaction surface and Modified Takeda hysteresis behaviour [5]. Strength degradation was assigned based on ductility in each direction. Control parameters for the concrete beam-column frame elements (Figure 1) were validated against experimental results.

For each of the two buildings designed to the different design philosophies, 2-dimensional (2D) computational models of the seismic frame (i.e. the frame not carrying gravity loads) were also constructed. The 2D models used the same hysteretic behaviour and parameters as the 3D models.

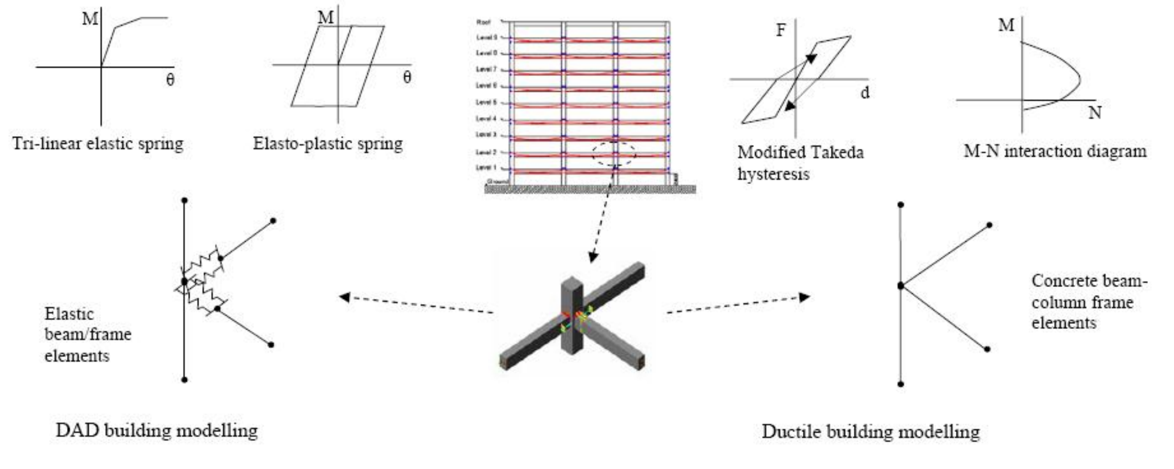

Figure 1: Computational modelling of buildings

3. IMPLEMENTATION OF IDA

To conduct IDA, first a computational model of the structure must be developed with appropriate material and geometric properties to encompass the full range of structural response from elastic behavior through to collapse. Secondly, a suite of earthquake records needs to be selected and all records should be scaled to several levels of Intensity Measure (IM). The structure is then subjected to each of the scaled earthquake records in the suite and a pre-determined Engineering Demand

Parameter (EDP) is monitored. These evaluations are carried out over a range of IM values to arrive at a matrix of data points on an IM vs EDP plot, representing the IDA curve. Many of the above steps can be automated using appropriate algorithms to select the scaling factors to apply to the records as well as running the computational simulations, therefore enabling the otherwise cumbersome process to be implemented without onerous time demands on users.

The selection of IM and EDP is by no means trivial and depends on the focus of the IDA. Current best practice suggests that for structures suitable to be modeled as a single degree of freedom (SDOF) system the 5% damped Spectral Acceleration (S_A) at the natural period of the structure is an appropriate IM (S_A=S_A[T, 5%]), as opposed to Peak Ground Acceleration (PGA). For multi-degree of freedom (MDOF) systems though, both PGA and S_A[T_1, 5%] (where T_1 is the period of the 1st mode response) have been used as the IM. When investigating the performance of the structure from a structural damage point of view, the maximum interstorey drift (θ_{max}) can be used as EDP as it relates well to joint rotations and both local and global collapse. On the other hand, when investigating non-structural damage, the horizontal floor accelerations would be more appropriate.

A suite of 20 bi-directional earthquake records described in Table 1 and obtained from the SAC steel project archive [6] were adopted. Each record shown in Table 1 represents one of the two orthogonal components of the ground motion. The suite of ground motions had a median source distance of 9.0 km, magnitude of 6.9, and spectral acceleration of 0.60g. Each of the records in Table 1 represents one of the two horizontal components of the earthquake records recorded in fault-normal and fault-parallel components. The components were then combined, rotated by 45 degrees, and resolved into orthogonal directions. This allows the records to be used for both bi-directional and uni-directional computation.

Table 1: Earthquake ground motion record properties

Ref	Event	Year	Station	M^{*1}	R^{*2} (km)	$S_A{}^{*3}$ (g)	Ref	Event	Year	Station	M^{*1}	R^{*3} (km)	$S_A{}^{*3}$ (g)
1	Imperial Valley	1940	El Centro	6.9	10	0.136	21	Kobe	1995	-	6.9	3.4	1.323
2	Imperial Valley	1940	El Centro	6.9	10	0.041	22	Kobe	1995	-	6.9	3.4	0.685
3	Imperial Valley	1979	Array #05	6.5	4.1	0.210	23	Loma Prieta	1989	-	7	3.5	0.550
4	Imperial Valley	1979	Array #05	6.5	4.1	0.180	24	Loma Prieta	1989	-	7	3.5	1.310
5	Imperial Valley	1979	Array #06	6.5	1.2	0.172	25	Northridge	1994	-	6.7	7.5	0.810
6	Imperial Valley	1979	Array #06	6.5	1.2	0.210	26	Northridge	1994	-	6.7	7.5	1.010
7	Landers	1992	Barstow	7.3	36	0.180	27	Northridge	1994	-	6.7	6.4	0.700
8	Landers	1992	Barstow	7.3	36	0.093	28	Northridge	1994	-	6.7	6.4	1.241
9	Landers	1992	Yermo	7.3	25	0.074	29	Tabas	1974	-	7.4	1.2	0.502
10	Landers	1992	Yermo	7.3	25	0.470	30	Tabas	1974	-	7.4	1.2	0.560
11	Loma Prieta	1989	Gilroy	7	12	0.170	31	Elysian Park	-	Simulated	7.1	17.5	1.330
12	Loma Prieta	1989	Gilroy	7	12	0.160	32	Elysian Park	-	Simulated	7.1	17.5	0.930
13	Northridge	1994	Newhall	6.7	6.7	0.190	33	Elysian Park	-	Simulated	7.1	10.7	1.010
14	Northridge	1994	Newhall	6.7	6.7	0.097	34	Elysian Park	-	Simulated	7.1	10.7	1.343
15	Northridge	1994	Rinaldi	6.7	7.5	0.083	35	Elysian Park	-	Simulated	7.1	11.2	1.694
16	Northridge	1994	Rinaldi	6.7	7.5	0.110	36	Elysian Park	-	Simulated	7.1	11.2	1.730
17	Northridge	1994	Sylmar	6.7	6.4	0.280	37	Palos Verdes	-	Simulated	7.1	1.5	0.850
18	Northridge	1994	Sylmar	6.7	6.4	0.021	38	Palos Verdes	-	Simulated	7.1	1.5	1.223
19	North Palm Springs	1986	North Palm Springs	6	6.7	0.310	39	Palos Verdes	-	Simulated	7.1	1.5	0.690
20	North Palm Springs	1986	North Palm Springs	6	6.7	0.210	40	Palos Verdes	-	Simulated	7.1	1.5	1.370

4. IDA RESULTS

4.1 UNI-DIRECTIONAL COMPARISON

To compare and contrast the behaviour of: (1) the 2D and 3D models; and (2) the DAD and ductile design philosophies, IDA was conducted. As an initial step to ensure compatibility between the 2D and 3D models 'uni-directional' IDA was conducted on all four models. For 3D models this involved applying the ground motion records in the direction of the seismic frame and monitoring the EDP in the same direction, without applying any ground motion in the orthogonal direction. For 2D models the ground motion was applied horizontally to the frame and the EDP monitored. Results showed that for each IM the corresponding EDP monitored was approximately the same for the 2D and 3D model for both design philosophies, hence verifying the compatibility of the models.

4.2 BI-DIRECTIONAL COMPARISON

Bi-directional interaction behaviour of the buildings was then considered by applying the bi-directional earthquakes to the 3D models and monitoring the EDP in the seismic frame direction (z direction). The component of the bi-directional earthquake aligned in the seismic frame direction was then applied to the 2D model and the EDP monitored. The IDA curves obtained are shown in Figure 2 for both the DAD and ductile buildings. The bi-directional response of the DAD building was not influenced much by the modelling; with the 3D IDA curves being within a few percent of the 2D IDA curves. The bi-directional interaction in the response of the ductile building is more apparent with EDP values typically 15% larger at the Design Basis Earthquake (DBE) intensity level (0.25g S_A) and 26% at the Maximum Considered Earthquake (MCE) intensity level (0.45g S_A). It is noted that the response of the 2D and 3D models of the ductile buildings are similar at low drift levels, but as the ground motion intensity increases, therefore increasing displacement demands, the drift response of the 3D building is more severe than that of the 2D building.

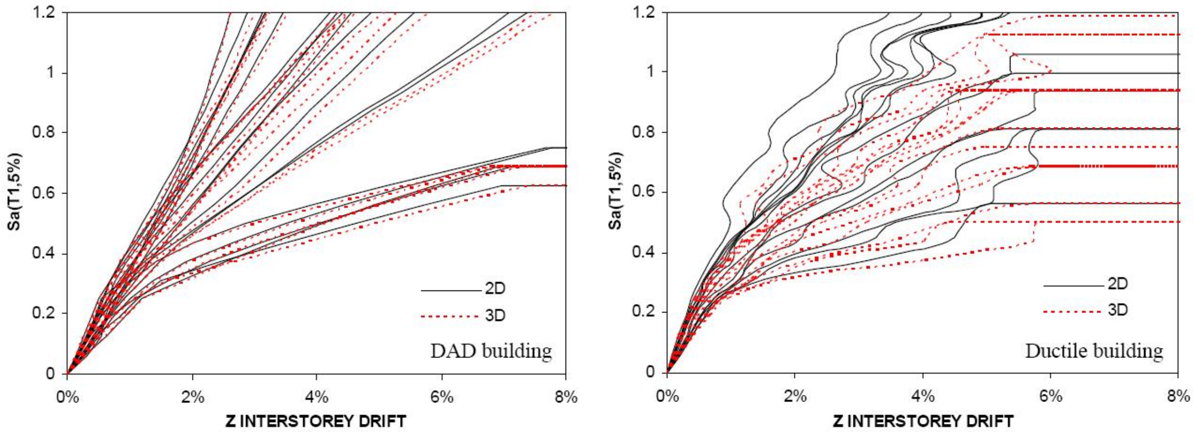

Figure 2: 2D-3D comparison of structures.

5. FRAGILITY CURVES

Fragility curves can be used to estimate the likelihood of user-defined Damage States (DS) being exceeded during an earthquake event. The first step is to define the EDP boundaries at which each DS will occur. Table 2 presents the damage states used in this research.

Table 1: Damage state limits

	Damage State	DAD Failure Mechanism	Ductile Failure Mechanism	Repair required	Outage expected	DAD Drift range	Ductile Drift range
DS1	None	Pre-yeild	None	None	None	0-0.01	0-0.005
DS2	Minor/ Slight	Dissipators yeild	Yeilding, Spalling	Replace Dissipators	<1 day	0.01-0.04	0.005-0.012
DS3	Moderate	Post-yeild, Spalling	Bar Buckling	Inspect, Re-tension, Patch	<3 days	0.04-0.1	0.012-0.025
DS4	Major	-	Degrading of strength	Rebuild Components	<3 months	-	0.025-0.046
DS5	Complete	Collapse	Collapse	Rebuild Structure	>3 months	>0.1	>0.046

The values of EDP at the DS boundaries are; (1) based on experimental results of a beam-column joint test from a prototype structure DAD building [7]; and (2) values adopted by Robertson [8] for a ductile equivalent of the same prototype building.

To compare the response of the two buildings designed to different philosophies in a probabilistic manner the 3D IDA curves and damage state boundaries were used to develop fragility curves.
A procedure called spectral reordering was used to reorder the EDP data points and assign probability ranges to different damage states. This was accomplished by sorting the twenty EDP's at each IM in descending order and defining a survival probability by the following formula:

$$S_i = 1 - \frac{i - 0.5}{n} \qquad (1)$$

where i = the rank of EDP's (i.e. 20 to 1) in descending order and n = number of earthquake records. The resulting plot of fragility curves (Figure 3) show the probability of exceeding a certain damage state for a given IM.

Fragility curves for the DAD building subjected to the suite of ground motion records show that at the DBE intensity level it is expected that no damage will occur (i.e. a probability of 0 that DS1 will be exceeded). At the MCE intensity level there is almost 100% probability of the DAD building response exceeding DS1 and almost no probability of exceeding DS2. At the DBE intensity level the response of the ductile building indicated a probability of 75% of exceeding DS1, and close to 0% of exceeding DS2. At the MCE intensity level the response of the ductile building indicated a probability of: (1) 100% of exceeding DS1; (2) 62.5% of exceeding DS2; (3) 20% of exceeding DS3; and (4) 6% of exceeding DS4 (i.e. collapse occurring).

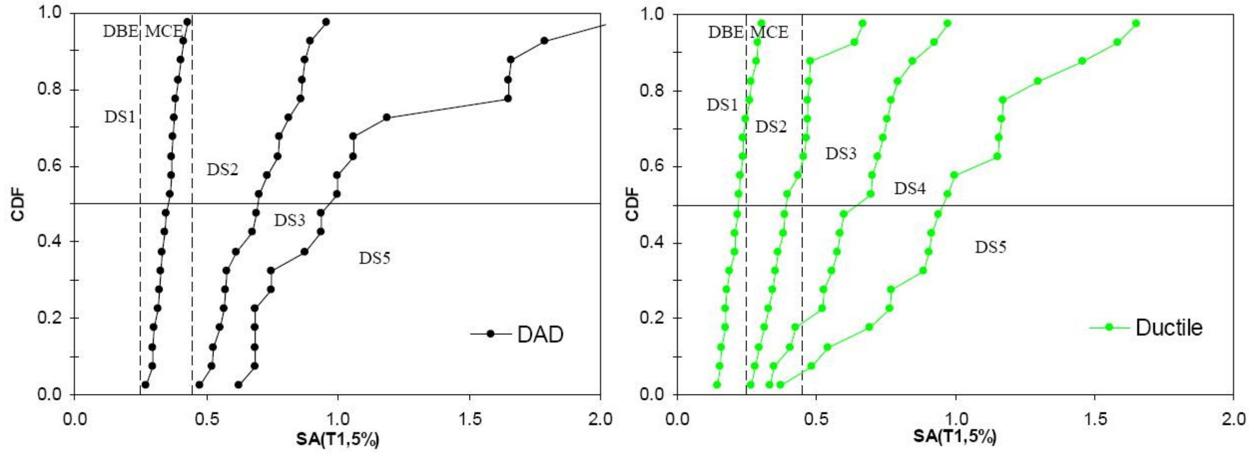

Figure 3: Fragility curve comparison of buildings designed to different design philosophies.

6. DISCUSSION

This research investigated the seismic response of two typical ten-storey buildings designed to conventional ductile design and Damage Avoidance Design (DAD), using both 2D and 3D computational models. Comparisons were made between: (1) the response of the buildings subjected to bi-directional ground motions in 3D and a single axis ground motion in 2D; and (2) the different responses of the two design philosophies. Data obtained in the form of IDA curves was then converted to Fragility curves using user-defined damage states based on previous experimental research observations.

From the aforementioned results it can be deduced that the simplification associated with considering the 2D response of a frame building for an earthquake ground motion can lead to conservative predictions of displacement demands. It can be qualitatively explained that bi-directional movement in the rocking joints present in the DAD building will cause rocking occurring on a member corner as opposed to a member face in unidirectional movement. This produces stress concentrations at the corner and leads to an increased flexibility of the joint region. The bi-directional rocking also causes an increase in the elongation of the unbonded prestressing tendons which correspondingly increases the joint restoring force. The above two phenomena oppose each other and therefore the overall response is similar to that observed in the 2D model. The increased response in the 3D ductile building compared to its 2D counterpart is mainly due to the interaction between the two directions causing non-linear (stiffness degradation) behaviour in elements. The global instability of both buildings causing collapse is primarily due to P-Δ effects.

A comparison between the responses of the buildings designed to different philosophies in terms of fragility curves showed that at the DBE intensity level no damage was expected in the DAD building, where as there was a 75% chance of yeilding of reinforcement and spalling of concrete in the ductile building. At the MCE intensity level the DAD building response indicated that supplemental energy dissipators would definitely (100% chance) be irreparably damaged but tendon yeilding would not occur (0% chance). At the MCE intensity level the ductile building response indicated a 63% chance of bar buckling, 20% chance of irreparable damage, and 6% chance of collapse occurring. These figures therefore statistically demonstrate the superior performance of the DAD philosophy over the current ductile/capacity design philosophy.

7. CONCLUSIONS

Based on the research conducted in this study the following conclusions can be made:

1. The non-linear bi-directional interaction apparent in the ductile building lead to increased interstorey drifts compared to those predicted by 2D modelling. The increased demands due to bi-directional interaction in the DAD building were negligible.

2. Fragility analysis of buildings conforming to the two different design philosophies indicate the superior performance at all intensity levels for the DAD details and construction compared to conventional capacity design with ductile details.

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